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# SECOND DRAFT OF CHAPTER 3
# DESIGN PARAMETERS

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# 3 DESIGN PARAMETERS

Applications of general concepts and methodologies of drainage design require the establishing of design parameters of the system under design. Such parameters specify the system performance with regard to various inputs. While the design concepts and methodologies are fairly general, the design parameters are site specific. Depending on the design objectives and the availability of supporting data, different design parameters may apply to each region, location, or project. Quite often, the basic drainage design parameters are established by a drainage authority (it can be a municipal, or a higher-level government) which issues drainage criteria and policies. In the absence of drainage criteria, the design parameters are established by the designer. For this purpose, the designer may have to examine various sources of supporting data, process such data, or even to undertake data collection programmes.

The major topics discussed in this Chapter include the planning horizon, the design period, catchment physical parameters, process parameters, rainfall data, quality parameters, and infiltration and inflow of extraneous flows into sewers.

## 3.1 Planning Horizon

The planning of drainage projects requires several types of information concerning not only the physical parameters of the system, but also the development trends, jurisdictional divisions, and financial arrangements. All the aforementioned factors will affect the planning process and the selection of the best alternative.

In drainage studies, the planning is best undertaken on the watershed basis. As a part of the watershed plan, a master drainage plan is prepared for the entire watershed, preferably at an early stage of the watershed development. The master drainage plan incorporates the whole

drainage system including the interrelation of the minor and major drainage systems.

Considering the relatively long design life of drainage structures, it is necessary to consider development factors in the watershed being analyzed. Such factors include the population trends, comprehensive plans prepared by planning and zoning groups, and locations of future roads, airports, industrial areas, etc., which may affect the drainage design. Because of large uncertainties in projections of development trends and the resulting uncertainties in the need and utilization of physical facilities, the planning process is more an art than science.

The planning is undertaken at two levels - the short-term and long-term planning. In the short-term plan, drainage facilities are planned on the basis of the proposed development plans for the period of 5 to 10 years. Such short-term plans must be compatible with the long-term plan. If needed, provisions are made for future expansion and extension of proposed drainage facilities. The long-term planning reflects the ultimate development of the watershed and is based on a longer planning period which may be as long as 20 to 30 years. To reduce uncertainties in the long-term plan, it is regularly updated (say every five years) to account for the actual development taking place and to reflect changes in technology and engineering practices.

The preparation of the master drainage plan starts with the identification of drainage related problems and definition of study objectives. These objectives generally include the abatement of local flooding and inconvenience (through implementation of minor drainage), abatement of downstream flood damages and threat to human life, protection of water supplies, and protection of water quality in the receiving waters. Study objectives are assigned various priorities and need to be accomplished under a given set of constraints which include natural, policy, regulation and cost constraints.

The next step consists in defining the drainage system components including the inputs, elements, and outputs. Examples of

inputs are design rainfall data and unit costs. System elements are established for the proposed development and for various drainage alternatives which are derived from considerations of various non-structural and structural measures employed in urban drainage design. Non-structural measures include land use policies, prohibition of flood plain occupancy, or the floodway-flood fringe concepts for flood plains. Structural measures include various drainage conduit and channel configurations, storage structures, diversion structures, channelization, dikes, and flood proffing. Finally, the system outputs are produced in the form of flood hydrographs and costs for various alternatives. These outputs are produced by means of various computational procedures which are described elsewhere in this manual.

All drainage alternatives are screened, compared, and the best alternative is selected on the basis of decision and evaluation criteria. The selected alternative is then implemented. Considering the dynamics of urban area development and changes in drainage technology, the master drainage plans should be regularly updated and modified, about once every five years.

## 3.2 Design Period for Drainage Projects

As discussed earlier in Chapter 2, the design period of drainage systems indicates the expected level of protection obtained under the proposed drainage systems. Depending on design objectives, this protection may apply to both flooding as well as water quality.

Sizing of storm sewers and drainage ditches is almost universally based on the concept of the design storm of a selected return period. In general, the selection of the design event return period is affected by the design life of structures involved, construction costs, and damage costs resulting from the system failure. Drainage structures are typically characterized by long design life and then the ideal design period should be the one for which the total annual drainage costs, defined as the sum of annual construction and drainage failure costs, are

minimal. The procedure which would be used to select such a least-cost design period is schematically shown in Fig. 1.

Most drainage projects do not warrant a detailed analysis of the construction and failure costs and the selection of the design period is largely based on experience. Even under those circumstances, the designer should consider the probability of the system failure and the consequences of the failure in terms of damage costs. The probability P of a design event with the return period T occurring at least once in N years may be expressed as

$$P = 1 - (1 - \frac{1}{T})^{N} \qquad (1)$$

Theoretical probabilities of failure for a given project life and design return period are shown in Fig. 2. Considering the design return period equal to the project life (T = N), the resulting probabilities of failure vary from 0.75 to 0.63, depending on the selected design period. Depending on the consequences of the drainage failure, such levels of risk may or may not be acceptable. To evaluate the acceptability of such risks, one has to differentiate between the minor and major drainage systems. The concept of minor/major drainage was introduced earlier in Chapter 2 and only a brief description is repeated below.

The minor drainage system comprises swales, street gutters, catchbasins, and storm sewers. It conveys runoff from frequent storms with return periods up to the design return period (e. g. 5 years). The minor system primarily reduces the incidence of inconvenience, caused by stormwater ponding, to both pedestrians and motorists. The consequences of the failure of the minor drainage system are not significant, provided that there is a properly functioning major drainage system. Therefore, the design periods for the minor system are relatively short, typically from 1 to 10 years. The shorter periods apply to residential areas, the longer periods apply to commercial and industrial areas where drainage failures could cause larger damage. The minor drainage system has to be

properly interfaced with the major drainage system to avoid overloading of minor drainage elements.

The major drainage system comprises natural streams and valleys as well as man-made drainage elements, such as streets, swales, channels and ponds. This system should accommodate the runoff from infrequent storms with long return periods (up to 100-years), or other critical regional events. A properly designed, constructed, and maintained major drainage system greatly reduces the risk of loss of life and property damage caused by flooding of urban areas. It should be emphasized that the major drainage system exists in nature regardless of whether it is identified as such and preserved during the urban development. It is the route which is followed by water during severe storms.

Consequences of the major drainage system failure are quite severe and may include appreciable flood damages and even the loss of life. Consequently, major drainage systems are designed at least for a 50-year storm, or more often, for a 100-year storm or a regional storm defined by a local drainage authority.

Two additional points should be made with regard to the design periods. Firstly, it should be emphasized that an identical design period does not have to be maintained throughout the drainage system. Some parts of the system, whose failure would be particularly costly, may be designed for longer design return periods than others. Secondly, the choice of the return period for the minor system may have some implications for the design period of the major system. By increasing the minor system design period and capacity, inflows to the major system will be reduced, thus reducing the costs of the major system.

A summary of typical design return periods which are used in the Canadian drainage practice (31) is given in Table 1.

Table 1 provides some guidance for the selection of design return periods. There are, however, some additional considerations to be made. In particular, longer design return periods are used for design of combined sewers to reduce the incidence of basement flooding, design of special structures (such as pumping stations), and for design of those

**TABLE 1. TYPICAL DESIGN RETURN PERIODS USED IN THE CANADIAN DRAINAGE PRACTICE (31)**

| Structure | Drainage System | Design Return Period (Years) |
|---|---|---|
| Culverts - Driveways | | 2*- 5 |
| - Local roads (span < 6 m) | | 10 |
| Storm sewers (No Surcharging) | Minor | |
| - Residential areas | | 2*- 5 |
| - High-value districts, commercial areas | | 5 - 10 |
| Elements of the major drainage system (swales, streets, channels, etc.) | Major | 100 |
| - Culverts and bridges | | 100 |

* Even shorter return periods (e.g., one year) can be used under different circumstances.

parts of the drainage system which are not susceptible to future relief (60).

The return periods given in Table 1 for storm sewers vary over a fairly wide range (2-10 years). It should be recognized, however, that the costs of storm sewer systems are little sensitive to the design return period. Several case studies indicated (10,60) that by doubling the design return period from 5 to 10 years, the costs increased by only 5-11%. Thus, it may be possible to provide a higher level of protection with a marginal increase in costs.

Finally, it should be mentioned that, as an alternative to the design event approach, a risk-based design approach has been proposed for urban drainage. Although such an approach is commonly used for design of large hydraulic structures, it has not gained much acceptance in drainage practice. The description of the risk-based design of storm sewers is given elsewhere (52, 68).

## 3.3 Catchment Physical Parameters

Physical characteristics of the catchment are required to establish drainage patterns in the catchment, linkages of various conveyance elements, and numerical values of process parameters. Because the computations often involve comparisons of pre-development and post-development flows, physical characteristics are needed for both states of the catchment development. The types of information required include catchment topography, drainage plans, and soil maps, as further discussed below.

Site maps are required showing the entire watershed and the area under consideration; details of topography including contours, watercourses, wooded areas, rock outcrops, and marshes; details of existing and proposed land use; and, the existing and proposed major drainage channels. Using such maps, it is possible to establish the study area drainage boundaries, general drainage patterns in the area, surface slopes, and the total catchment area. The total catchment area is further subdivided into impervious and pervious parts. In the impervious part, an impermeable surface cover does not allow infiltration of water into the ground. Typical examples of impervious elements include paved streets, sidewalks and yards; roofs; and, rock outcrops. The catchment imperviousness is usually expressed as the ratio of the impervious area to the total catchment area.

The impervious area can be further divided into the effective and noneffective areas. The former one refers to the impervious elements which drain directy into transport elements (servers). The latter areas

represent impervious elements which drain onto pervious elements, or have no drainage outlet.

The pervious area is sometimes divided into the contributing and noncontributing parts. The contributing areas drain into the drainage transport system; the noncontributing part has no drainage outlets.

All the above listed areas need to be delineated and characterized in terms of the area, plan geometry, and surface slopes. For various reasons, the catchment studied is usually subdivided into a number of subcatchments and then the above information needs to be determined for each of these subcatchments.

All the existing drainage channels and conduits in the catchment need to be characterized in terms of the cross-sections, slopes, lengths, linkages, and installations.

## 3.4 Process Parameters

In deterministic hydrological calculations, it is assumed that the relationships between many interactive factors affecting the water balance can be defined analytically and the numeric values used to quantify the movement and storage of water are called parameters. Although such parameters can be quite numerous, only a few of these are used in typical urban flood computations. Such parameters include infiltration rates, depression storage, flow roughness coefficients, and runoff coefficients. Numeric values of these parameters are obtained by field measurements, calibrations, or most frequently, by tranposition from other similar catchments. The discussion that follows concentrates on urban applications.

Infiltration Rates A proper presentation of infiltration in a catchment is a complex task which often extends beyond the scope of hydrologic concepts and methods employed in urban drainage design. Most often, infiltration rates are evaluated from soil physical properties which are

obtained from soil maps. For this purpose, soils are classified according to their drainage properties and the corresponding infiltration rates are selected from the literature. In urban applications in North America, the U.S. Soil Conservation Service (SCS) classification of soils into four hydrologic groups A-D is fairly common (56). The knowledge of the hydrologic soil group and initial soil moisture conditions is sufficient to estimate the infiltration rates based on the SCS method and to select the appropriate parameter values for Horton's equation, or to select a runoff coefficient for pervious areas.

In physically based approaches, such as those described by the Holtan and Green-Ampt equations, more information on soils is required. In particular, the division of the soil profile into various horizons needs to be known, together with the soil porosity and various types of water storage (see Chapter 6).

Simplified approaches to infiltration may be acceptable for urban catchments in which the generation of runoff is mainly controlled by impervious elements. In other cases, the most comprehensive approach which can be supported by the available data should be used. Such approaches are generally physically based and involve continuous simulation of water storage in soils.

Depression storage Depression storage accounts for rainwater trapped on the catchment surface in minute depressions, that does not run off or infiltrate into the soil. Generally, it represents a combination of several hydrologic abstractions. For urban floods, the depression storage is of secondary importance. Typical values are 1.6 mm for impervious areas and 5 mm for pervious areas.

Roughness of transport elements In flow routing calculations, the roughness of individual elements needs to be determined. In urban applications, this is estimated most often by means of the Manning roughness coefficient, n. For concrete conduits, concrete-lined channels, and impervious overland flow planes, the value of n = 0.013 is

widely used. Manning's n values for overland flow on grassed areas vary from 0.2 to 0.35. Extensive listings of n-values for various conveyance elements can be found in refs. 20 and 28.

## 3.5 Rainfall Data for Drainage Design

Rainfall data represent the most important input to runoff computations. Consequently, much attention in drainage design focuses on the selection and processing of rainfall data. Various procedures used for this purpose are discussed in this section. The main topics discussed include the sources of rainfall data, the point vs. areal rainfall, frequency analysis of point rainfall, and design storms.

### 3.5.1 Sources of rainfall data

Rainfall data are typically collected by national hydrometeorological agencies (or similar agencies), which have been established in most countries, in a network of stations distributed throughout the country. Where the density of such a network and the locations of individual stations do not meet the needs of drainage design, the local drainage authority may have to establish its own monitoring network using the information given in the accompanying UNESCO manual (54). In fact, it is quite common in some countries that large metropolitan municipalities operate their own network of raingauges.

Rainfall data are distributed in various forms, most often as records of hourly rainfall amounts, or one can obtain copies of actual raingauge records. For drainage design, hourly rainfall data may be adequate at the planning stage. For detailed design, however, short time intervals (5-15 minutes) intensities are needed. Such intensities are obtained by analysis of raingauge records. The shorter intervals (5 min.) apply to small areas ~100 ha), the longer intervals apply to larger areas (> 1000 ha).

In the absence of local rainfall data, such data may be obtained through tranposition or interpolation from other areas. Although individual raingauge records may not be representative of rainfall over large areas, the general characteristics of rainfall are quite conservative in space and vary smoothly in non-mountainous areas (42). Thus parameters like frequency of rainfall above a given value may be transposed or interpolated to nearby areas without data. Ideally, this task is carried out by a national hydrometeorological service which distributes standardized rainfall data for various climatic regions of the country.

Another important consideration in the evaluation of existing rainfall records is the record length. Excessive extrapolations of limited data, leading to large uncertainities, should be avoided. For example, the Canadian Atmospheric Environment Service (AES) undertakes annual maximum rainfall analysis only for records longer than seven years and the data are extrapolated for return periods up to twice as long as the record length (42).

### 3.5.2 Point vs areal rainfall

In small-scale drainage projects, one may assume the validity of the point rainfall for the entire drainage area. It has been suggested that a single gauge be taken as representative of the rainfall over small basins up to 25 $km^2$ in area (66). In urban areas which are generally characterized by fast response, a more stringent criterion may be desirable. For areas larger than 25 $km^2$, the areal rainfall will be smaller than the point rainfall because of decrease in rainfall intensity away from the storm centre. To account for this decrease, reduction factors which depend on the basin area are applied to the point rainfall to obtain the average areal rainfall. Reduction factors recommended by WMO (66) are shown in Fig. 3.

### 3.5.3 Frequency analysis of point rainfall

Frequency analysis is used to derive the Intensity-Duration-Frequency (I-D-F) curves which give the probability of exceedance of rainfall of certain duration and intensity. In drainage design, the I-D-F curves represent the most important form of rainfall data. These curves are used in computations of runoff peaks by the Rational Method, or they can be used to develop certain types of design storms for runoff simulations. Because of the importance of the I-D-F curves, a detailed description of the development of I-D-F curves is given below.

The preparation of I-D-F curves consists of two interrelated tasks - abstraction of data from the rainfall record and the frequency analysis of the abstracted data.

For frequency analysis, two types of rainfall data series may be used. Preferably, an annual series consisting of annual rainfall maxima for various durations, typically from five minutes to 24 hours, is used. Such an analysis is expedient (the data series is rather limited), there is a theoretical basis for extrapolaion of the annual series beyond the record length, and the results of the annual series analysis can be modified to make them compatible to those obtained from the partial duration series by using the coefficients given in Table 2.

**TABLE 2. RELATIONSHIP BETWEEN ANNUAL AND PARTIAL SERIES (42)**

| | Return Period (years) | | | |
|---|---|---|---|---|
| | 2 | 5 | 10 | >10 |
| (Partial series value)/(Annual series value) | 1.13 | 1.04 | 1.01 | 1.00 |

The other type of rainfall data series is referred to as the partial duration series and contains all large rainfall amounts above certain thresholds selected for individual durations. Thus for individual years, not only the annual maximum but possibly also other values are included in the analysis. The partial duration analysis appears to be preferable for short records, or where return periods shorter than two years are of interest.

In the frequency analysis, the abstracted large rainfall amounts are ranked and assigned probabilities using various plotting position formulas (8). For the annual series, the Weibull's formula is generally recommended (8) in the following form:

$$P = \frac{m}{N+1} \qquad (2)$$

where P is the probability, m, is the rank (the largest value corresponds to m = 1), and N is the record length in years.

For a partial duration series, a different position formula is sometimes used in the following form (8):

$$P = \frac{m}{N} \qquad (3)$$

where N means the number of items. It should be emphasized that both equations (2) and (3) give almost identical results in the middle of the distributions but differ near the tails of the distributions.

Having established the plotting positions, the abstracted rainfall amounts can be plotted either on the Gumbel probability paper (the annual series), or on a semi-logarithmic paper (4,6). In the latter case, the probabilities (or the return periods) are plotted on the x-axis in the logarithmic scale and the rainfall amounts (or intensities) on the y-axis in the linear scale. Finally, the experimental data are approximated by a straight line which is used for further extrapolations

or interpolations. The line fitting is accomplished either graphically or mathematically. In the later case, there is no need to plot the data. One of the mathematical fitting procedures is demonstrated in the example given below.

Example of Intensity-Duration-Frequency Analysis

The following example of the IDF analysis has been adopted from a document of the(Canadian) Atmospheric Environment Service (AES) (42). Other agencies may recommend different procedures of equal merit.

Many attempts have been made to derive theoretical or empirical relationships which would describe well the probability distributions of hydrological variables. Such procedures are described in various standard texts on hydrology (39, 49). The discussion below is limited to the asymptotic extremal distribution, also known as the Fisher-Tippet Type I distribution, which can be applied using the procedure developed by Gumbel (17).

It can be shown that the hydrological frequency function for extreme rainfall can be described as

$$X_T = \overline{X} + KS_x \qquad (4)$$

where $\overline{X}$ is the mean and $S_x$ is the standard deviation of the sample of extreme rainfall, and $X_T$ is the rainfall amount which is equalled or exceeded on an average once in T years. K is the frequency factor which can be expressed in terms of the return period T and the number of years of record, N. The values of K can be obtained from standard tables, such as the one reproduced in Table 3 (42).

Equation 4 is used to calculate the rainfall amount $X_T$ of a return period T. For that purpose, the mean and standard deviation of the annual series are calculated first and the value of K, for the return

TABLE 3. K Factors for the Gumbel Distribution (After Ref. 42)

| N (yr) | K Factors — Return Period T (years) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 2 | 5 | 10 | 15 | 20 | 25 | 50 | 100 |
| 5 | -0.116 | 1.313 | 2.760 | 2.794 | 3.168 | 3.456 | 4.343 | 5.224 |
| 6 | -0.122 | 1.229 | 2.124 | 2.629 | 2.982 | 3.254 | 4.093 | 4.925 |
| 7 | -0.127 | 1.169 | 2.026 | 2.510 | 2.849 | 3.110 | 3.914 | 4.712 |
| 8 | -0.130 | 1.123 | 1.953 | 2.421 | 2.749 | 3.001 | 3.779 | 4.551 |
| 9 | -0.133 | 1.087 | 1.895 | 2.351 | 2.670 | 2.916 | 3.673 | 4.425 |
| 10 | -0.136 | 1.058 | 1.848 | 2.294 | 2.606 | 2.847 | 3.587 | 4.323 |
| 11 | -0.138 | 1.034 | 1.809 | 2.247 | 2.553 | 2.789 | 3.516 | 4.238 |
| 12 | -0.139 | 1.013 | 1.777 | 2.207 | 2.509 | 2.741 | 3.456 | 4.166 |
| 13 | -0.141 | 0.996 | 1.748 | 2.173 | 2.470 | 2.699 | 3.405 | 4.105 |
| 14 | -0.142 | 0.981 | 1.724 | 2.143 | 2.437 | 2.663 | 3.360 | 4.052 |
| 15 | -0.143 | 0.967 | 1.703 | 2.117 | 2.408 | 2.632 | 3.321 | 4.005 |
| 16 | -0.144 | 0.955 | 1.683 | 2.094 | 2.382 | 2.603 | 3.286 | 3.963 |
| 17 | -0.145 | 0.945 | 1.666 | 2.074 | 2.359 | 2.578 | 3.255 | 3.926 |
| 18 | -0.146 | 0.935 | 1.651 | 2.055 | 2.338 | 2.556 | 3.227 | 3.893 |
| 19 | -0.147 | 0.927 | 1.637 | 2.038 | 2.319 | 2.535 | 3.202 | 3.863 |
| 20 | -0.148 | 0.919 | 1.625 | 2.023 | 2.302 | 2.517 | 3.179 | 3.836 |
| 21 | -0.148 | 0.911 | 1.613 | 2.009 | 2.286 | 2.500 | 3.158 | 3.811 |
| 22 | -0.149 | 0.905 | 1.603 | 1.996 | 2.272 | 2.484 | 3.138 | 3.788 |
| 23 | -0.150 | 0.899 | 1.593 | 1.984 | 2.259 | 2.470 | 3.120 | 3.766 |
| 24 | -0.150 | 0.893 | 1.584 | 1.974 | 2.246 | 2.457 | 3.104 | 3.747 |
| 25 | -0.151 | 0.888 | 1.575 | 1.963 | 2.235 | 2.444 | 3.089 | 3.728 |
| 26 | -0.151 | 0.883 | 1.568 | 1.954 | 2.224 | 2.433 | 3.074 | 3.711 |
| 27 | -0.151 | 0.878 | 1.560 | 1.945 | 2.214 | 2.422 | 3.061 | 3.695 |
| 28 | -0.152 | 0.874 | 1.553 | 1.937 | 2.205 | 2.412 | 3.048 | 3.681 |
| 29 | -0.152 | 0.870 | 1.547 | 1.929 | 2.196 | 2.402 | 3.037 | 3.667 |
| 30 | -0.153 | 0.866 | 1.541 | 1.922 | 2.188 | 2.393 | 3.026 | 3.653 |
| 31 | -0.153 | 0.863 | 1.535 | 1.915 | 2.180 | 2.385 | 3.015 | 3.641 |
| 32 | -0.153 | 0.859 | 1.530 | 1.908 | 2.173 | 2.377 | 3.005 | 3.629 |
| 33 | -0.153 | 0.856 | 1.525 | 1.902 | 2.166 | 2.369 | 2.996 | 3.618 |
| 34 | -0.154 | 0.853 | 1.520 | 1.896 | 2.159 | 2.362 | 2.987 | 3.608 |
| 35 | -0.154 | 0.850 | 1.515 | 1.891 | 2.153 | 2.356 | 2.979 | 3.598 |
| 36 | -0.154 | 0.848 | 1.511 | 1.885 | 2.147 | 2.349 | 2.971 | 3.588 |
| 37 | -0.155 | 0.845 | 1.507 | 1.880 | 2.142 | 2.343 | 2.963 | 3.579 |
| 38 | -0.155 | 0.843 | 1.503 | 1.875 | 2.136 | 2.337 | 2.956 | 3.570 |
| 39 | -0.155 | 0.840 | 1.499 | 1.871 | 2.131 | 2.332 | 2.949 | 3.562 |
| 40 | -0.155 | 0.838 | 1.495 | 1.866 | 2.126 | 2.326 | 2.943 | 3.554 |
| 41 | -0.155 | 0.836 | 1.492 | 1.862 | 2.121 | 2.321 | 2.936 | 3.547 |
| 42 | -0.156 | 0.834 | 1.489 | 1.858 | 2.117 | 2.316 | 2.930 | 3.539 |
| 43 | -0.156 | 0.832 | 1.485 | 1.854 | 2.113 | 2.311 | 2.924 | 3.533 |
| 44 | -0.156 | 0.830 | 1.482 | 1.851 | 2.108 | 2.307 | 2.919 | 3.526 |
| 45 | -0.156 | 0.828 | 1.479 | 1.847 | 2.104 | 2.303 | 2.913 | 3.519 |
| 46 | -0.156 | 0.826 | 1.477 | 1.844 | 2.101 | 2.298 | 2.908 | 3.513 |
| 47 | -0.156 | 0.824 | 1.474 | 1.840 | 2.097 | 2.294 | 2.903 | 3.507 |
| 48 | -0.157 | 0.823 | 1.471 | 1.837 | 2.093 | 2.291 | 2.898 | 3.502 |
| 49 | -0.157 | 0.821 | 1.469 | 1.834 | 2.090 | 2.287 | 2.894 | 3.496 |
| 50 | -0.157 | 0.820 | 1.466 | 1.831 | 2.086 | 2.283 | 2.889 | 3.491 |
| 51 | -0.157 | 0.818 | 1.464 | 1.828 | 2.083 | 2.280 | 2.885 | 3.486 |
| 52 | -0.157 | 0.817 | 1.462 | 1.825 | 2.080 | 2.276 | 2.881 | 3.481 |
| 53 | -0.157 | 0.815 | 1.459 | 1.823 | 2.077 | 2.273 | 2.877 | 3.476 |
| 54 | -0.157 | 0.814 | 1.457 | 1.820 | 2.074 | 2.270 | 2.873 | 3.471 |
| 55 | -0.157 | 0.813 | 1.455 | 1.818 | 2.071 | 2.267 | 2.869 | 3.467 |
| 56 | -0.158 | 0.812 | 1.453 | 1.815 | 2.069 | 2.264 | 2.865 | 3.462 |
| 57 | -0.158 | 0.810 | 1.451 | 1.813 | 2.066 | 2.261 | 2.862 | 3.458 |
| 58 | -0.158 | 0.809 | 1.449 | 1.811 | 2.063 | 2.258 | 2.858 | 3.454 |
| 59 | -0.158 | 0.808 | 1.448 | 1.808 | 2.061 | 2.256 | 2.855 | 3.450 |
| 60 | -0.158 | 0.807 | 1.446 | 1.806 | 2.059 | 2.253 | 2.852 | 3.446 |
| 61 | -0.158 | 0.806 | 1.444 | 1.804 | 2.056 | 2.250 | 2.849 | 3.442 |
| 62 | -0.158 | 0.805 | 1.442 | 1.802 | 2.054 | 2.248 | 2.846 | 3.439 |
| 63 | -0.158 | 0.804 | 1.441 | 1.800 | 2.052 | 2.246 | 2.843 | 3.435 |
| 64 | -0.158 | 0.803 | 1.439 | 1.798 | 2.050 | 2.243 | 2.840 | 3.432 |
| 65 | -0.158 | 0.802 | 1.438 | 1.796 | 2.047 | 2.241 | 2.837 | 3.428 |
| 66 | -0.159 | 0.801 | 1.436 | 1.794 | 2.045 | 2.239 | 2.834 | 3.425 |
| 67 | -0.159 | 0.800 | 1.435 | 1.793 | 2.043 | 2.237 | 2.831 | 3.422 |
| 68 | -0.159 | 0.799 | 1.433 | 1.791 | 2.041 | 2.234 | 2.829 | 3.419 |
| 69 | -0.159 | 0.798 | 1.432 | 1.789 | 2.040 | 2.232 | 2.826 | 3.416 |
| 70 | -0.159 | 0.797 | 1.430 | 1.788 | 2.038 | 2.230 | 2.824 | 3.413 |
| 71 | -0.159 | 0.797 | 1.429 | 1.786 | 2.036 | 2.228 | 2.821 | 3.410 |
| 72 | -0.159 | 0.796 | 1.428 | 1.784 | 2.034 | 2.227 | 2.819 | 3.407 |
| 73 | -0.159 | 0.795 | 1.427 | 1.783 | 2.032 | 2.225 | 2.817 | 3.404 |
| 74 | -0.159 | 0.794 | 1.425 | 1.781 | 2.031 | 2.223 | 2.814 | 3.402 |
| 75 | -0.159 | 0.793 | 1.424 | 1.780 | 2.029 | 2.221 | 2.812 | 3.399 |
| 76 | -0.159 | 0.793 | 1.423 | 1.779 | 2.028 | 2.219 | 2.810 | 3.397 |
| 77 | -0.159 | 0.792 | 1.422 | 1.777 | 2.026 | 2.218 | 2.808 | 3.394 |
| 78 | -0.159 | 0.791 | 1.421 | 1.776 | 2.024 | 2.216 | 2.806 | 3.392 |
| 79 | -0.159 | 0.791 | 1.420 | 1.774 | 2.023 | 2.214 | 2.804 | 3.389 |
| 80 | -0.159 | 0.790 | 1.419 | 1.773 | 2.021 | 2.213 | 2.802 | 3.387 |
| 81 | -0.160 | 0.789 | 1.417 | 1.772 | 2.020 | 2.211 | 2.800 | 3.385 |
| 82 | -0.160 | 0.789 | 1.416 | 1.771 | 2.019 | 2.210 | 2.798 | 3.382 |
| 83 | -0.160 | 0.788 | 1.415 | 1.769 | 2.017 | 2.208 | 2.796 | 3.380 |
| 84 | -0.160 | 0.787 | 1.414 | 1.768 | 2.016 | 2.207 | 2.795 | 3.378 |
| 85 | -0.160 | 0.787 | 1.413 | 1.767 | 2.015 | 2.205 | 2.793 | 3.376 |
| 86 | -0.160 | 0.786 | 1.413 | 1.766 | 2.013 | 2.204 | 2.791 | 3.374 |
| 87 | -0.160 | 0.786 | 1.412 | 1.765 | 2.012 | 2.203 | 2.789 | 3.372 |
| 88 | -0.160 | 0.785 | 1.411 | 1.764 | 2.011 | 2.201 | 2.788 | 3.370 |
| 89 | -0.160 | 0.785 | 1.410 | 1.763 | 2.010 | 2.200 | 2.786 | 3.368 |
| 90 | -0.160 | 0.784 | 1.409 | 1.762 | 2.008 | 2.199 | 2.784 | 3.366 |
| 91 | -0.160 | 0.783 | 1.408 | 1.761 | 2.007 | 2.197 | 2.783 | 3.364 |
| 92 | -0.160 | 0.783 | 1.407 | 1.759 | 2.006 | 2.196 | 2.781 | 3.362 |
| 93 | -0.160 | 0.782 | 1.406 | 1.758 | 2.005 | 2.195 | 2.780 | 3.360 |
| 94 | -0.160 | 0.782 | 1.406 | 1.758 | 2.004 | 2.194 | 2.778 | 3.359 |
| 95 | -0.160 | 0.781 | 1.405 | 1.757 | 2.003 | 2.193 | 2.777 | 3.357 |
| 96 | -0.160 | 0.781 | 1.404 | 1.756 | 2.002 | 2.191 | 2.775 | 3.355 |
| 97 | -0.160 | 0.780 | 1.403 | 1.755 | 2.001 | 2.190 | 2.774 | 3.354 |
| 98 | -0.160 | 0.780 | 1.403 | 1.754 | 2.000 | 2.189 | 2.773 | 3.352 |
| 99 | -0.160 | 0.780 | 1.402 | 1.753 | 1.999 | 2.188 | 2.771 | 3.350 |
| 100 | -0.160 | 0.779 | 1.401 | 1.752 | 1.998 | 2.187 | 2.770 | 3.349 |

period T and the record length N, is read from Table 3. If one needs to determine the return period of a rainfall value $X_T$, equation 4 is used to calculate K, and the corresponding T is read from Table 3 (i.e. for given $\bar{X}$, $S_X$ and N). The return period T can also be determined from frequency plots on extreme (Gumbel) probability paper. If the plotting positions are calculated from eq. (2), then equation 4 plots as a straight line on the Gumbel probability paper.

An example of the computation of the I-D-F curves has been adopted from an AES report (42) and is presented in Table 4. Further explanations follow.

In the upper part of Table 4, annual maximum rainfall amounts are listed for nine durations from five minutes to 24 hours, and the record length of 12 years. For each duration, the mean and standard deviation were calculated. Subsequently, the rainfall amounts for various durations and return periods were calculated from Equation 4 by substituting for $\bar{X}$, $S_X$ and K. The values of K were determined from Table 3 for N = 12 and return periods varying from 2 to 100 years.

In the bottom part of the table, rainfall amounts were converted into intensities ($i = h/\Delta t$, where i is the rainfall intensity, h is the rainfall amount, and $\Delta t$ is the corresponding duration). The calculated rainfall intensities were plotted in Fig. 4 and approximated by smooth curves. For simplicity, the curves for T = 2 and 5 were not converted to partial series results (see the coefficients in Table 2) and, although the record length was adequate only for return periods up to 25 years, two I-D-F curves for T = 50 and 100 were also included.

If no short-duration rainfall data are available and data transposition or interpolation is infeasible, it is sometimes possible to estimate the short-duration rainfall statistics from other available data. Such data include the mean annual precipitation, the mean annual number of days with rain, and the mean annual number of thunderstorm days. WMO (66) gives a summary of such techniques. When applying these

**TABLE 4. EXAMPLE OF CALCULATION OF INTENSITY-DURATION-FREQUENCY CURVES**

| Year | Durations | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 5 min | 10 min | 15 min | 30 min | 1 hr | 2 hr | 6 hr | 12 hr | 24 h |
| | Rainfall Amounts (mm) | | | | | | | | |
| 1968 | 5.6 | 7.6 | 7.9 | 7.9 | 9.9 | 16.5 | 30.5 | 36.1 | 36. |
| 1969 | 5.1 | 9.1 | 13.7 | 15.5 | 15.5 | 15.5 | 34.3 | 41.1 | 41. |
| 1970 | 3.8 | 5.6 | 6.6 | 8.4 | 10.9 | 19.6 | 22.6 | 24.1 | 31. |
| 1971 | 2.5 | 3.6 | 4.8 | 7.6 | 11.7 | 18.8 | 20.8 | 21.1 | 23. |
| 1972 | 7.6 | 7.9 | 8.4 | 9.7 | 12.2 | 17.0 | 25.7 | 31.0 | 39. |
| 1973 | 6.9 | 8.9 | 10.4 | 11.7 | 16.8 | 23.6 | 27.9 | 30.0 | 36. |
| 1974 | 2.3 | 3.3 | 4.3 | 5.3 | 6.3 | 9.1 | 23.4 | 29.5 | 41. |
| 1975 | 6.3 | 9.7 | 10.2 | 10.9 | 15.5 | 16.8 | 28.4 | 28.4 | 33. |
| 1976 | 2.8 | 3.8 | 5.6 | 8.4 | 10.2 | 13.5 | 19.3 | 20.3 | 20. |
| 1977 | 6.3 | 6.3 | 6.3 | 9.9 | 14.0 | 15.5 | 18.3 | 33.3 | 33. |
| 1978 | 14.5 | 15.6 | 17.8 | 24.8 | 27.3 | 27.3 | 27.3 | 36.3 | 50. |
| 1979 | 3.9 | 4.8 | 5.4 | 7.5 | 8.1 | 13.4 | 15.8 | 22.2 | 26. |
| Mean Extreme (N=12) | 5.6 | 7.2 | 8.4 | 10.6 | 13.2 | 17.2 | 24.5 | 29.4 | 34. |
| Standard Deviation | 3.2 | 3.3 | 3.9 | 4.9 | 5.2 | 4.6 | 5.2 | 6.3 | 8. |
| Return Period [yr] | Rainfall Amounts (mm) | | | | | | | | |
| 2 | 5.2 | 6.7 | 7.9 | 9.9 | 12.5 | 16.6 | 23.8 | 28.6 | 33. |
| 5 | 8.8 | 10.6 | 12.4 | 15.6 | 18.5 | 21.9 | 29.8 | 35.9 | 42. |
| 10 | 11.3 | 13.1 | 15.3 | 19.4 | 22.4 | 25.4 | 33.8 | 40.7 | 49. |
| 25 | 14.3 | 16.3 | 19.0 | 24.2 | 27.5 | 29.8 | 38.9 | 46.8 | 57. |
| 50* | 16.6 | 18.7 | 21.8 | 27.7 | 31.2 | 33.0 | 42.6 | 51.3 | 63. |
| 100* | 18.8 | 21.0 | 24.5 | 31.2 | 34.9 | 36.3 | 46.3 | 55.8 | 68. |
| Return Period | Rainfall Intensities (mm/hr) | | | | | | | | |
| 2 | 62.4 | 40.2 | 31.6 | 19.8 | 12.5 | 8.3 | 4.0 | 2.4 | 1. |
| 5 | 105.6 | 63.6 | 49.6 | 31.2 | 18.5 | 11.0 | 5.0 | 3.0 | 1. |
| 10 | 135.6 | 78.6 | 61.2 | 38.8 | 22.4 | 12.7 | 5.6 | 3.4 | 2. |
| 25 | 171.6 | 97.8 | 76.0 | 48.4 | 27.5 | 14.9 | 6.5 | 3.9 | 2. |
| 50* | 199.2 | 112.2 | 87.2 | 55.4 | 31.2 | 16.5 | 7.1 | 4.3 | 2. |
| 100* | 225.6 | 126.0 | 98.0 | 62.4 | 34.9 | 18.2 | 7.7 | 4.7 | 2. |

* Extrapolations to these long return periods were included only for completeness.

techniques, one should be aware of possible large uncertainties in the results.

I-D-F curves are commonly determined for all stations in the network and mapped. Such maps are then used to interpolate the I-D-F statistics to locations without data.

I-D-F curves are used extensively in urban drainage design. Their primary use is to provide a rainfall input for the Rational Method or similar empirical formulas. Such an input is typically a constant-intensity rainfall of a certain duration and return period. For example, if the time of concentration is one hour (= duration), and the design return period is ten years, one obtains the design rainfall from Fig. 3. as i = 227 mm/hr. This design value is valid only for the location for which the I-D-F curves in Fig. 4 were produced.

Another use of I-D-F curves is for the development of design storms, as discussed in the next section.

### 3.5.4 Design storms

The design storm concept was introduced into urban drainage design about 25 years ago to replace constant-intensity design rainfalls which were adequate only for steady-state computations of runoff peak flows. As runoff computations progressed from empirical formulas for the peak flow to the hydrologic synthesis based on time-varying losses, it became necessary to consider time-varying rainfall inputs referred to as design storms. Such storms are typically derived by synthesizing generalizations from a large number of actual events.

In the design storm concept, an assumption is made that the return period of the calculated runoff event is identical to that of the design storm. Although this assumption and some other aspects of design storms have been criticized by many researchers (32, 35, 46), the design storm concept is widely used in the engineering practice. Design storms can be developed expediently from local rainfall data and used

economically in conjunction with event runoff models. Common design storms which are discussed below have been found adequate (3) for sizing of storm sewers and drainage ditches. Their application, however, should not be extended to more complex design aspects, such as storage design, comparisons of predevelopment and postdevelopment flows, water quality considerations, and real-time operation of combined sewer systems. For these complex problems, one may have to use actual rainfall data in conjunction with continuous simulation models, or to develop special rainfall inputs.

Design storms and their ability to produce runoff peaks are difficult to evaluate. Attempts have been made to accomplish such evaluations through comparisons of runoff peak frequency curves produced from simulations with actual and design storms (3, 32). The results of such studies are inconclusive and depend on the simulation model as well as on detailed simulation procedures. It appears that in many situations both synthetic and actual design storms produce comparable results which are adequate for sizing of sewers (3). These storms are sufficiently characterized by the total rainfall amount, temporal rainfall distribution and some indication of the antecedent conditions.

### 3.5.4.1 Design storms for sewer sizing

A number of design storms for sewer sizing have been proposed by various investigators (5, 11, 25, 26, 38, 43, 47, 48, 53, 67). In the absence of systematic evaluation of various design storms, no particular storm can be recommended as the best one. Consequently, the following discussion focuses on basic parameters of design storms and on approaches to estimating such parameters. In general, the design storms discussed in this section are fully characterized by the duration, the total rainfall amount, the maximum rainfall intensity of a certain short duration, the timing of peak intensity, and the temporal storm rainfall distribution. A discussion of individual storm parameters follows. Whenever appropriate, a reference is made to the existing design storms.

The storm duration can be taken as a fixed time period (53), or it can be derived from the statistical analysis of actual storms (11), or from the time of concentration of the area under design (25,38). In the last case, the time of concentration is defined as the travel time from the most remote point. The recommended storm durations vary from one (25) to three times (38) the time of concentration. It would appear that the best estimates of the storm duration should consider actual storm durations as well as the catchment concentration time. The selection of the storm duration is particularly important for those synthetic storms whose peak intensity is affected by the storm duration (53).

The total storm rainfall depth is determined from I-D-F curves for a selected storm duration and a return period. For example, considering a 6-hour 50-year storm, one obtains from Fig. 4, H = 6x7.5 = 45 mm. This rainfall depth is then distributed over the total storm duration.

The peak intensity of a particular storm hyetograph is determined for a certain short duration which usualy coincides with the computational time step used in runoff computations. Typically, this duration is equal to the shortest duration used in I-D-F curves - five minutes. The peak intensity is determined either by applying the temporal distribution to the storm rainfall (38, 53), or from I-D-F curves (25). The latter approach is used in the Chicago storm which is based on an assumption that, for a selected return period, all maximum intensities for durations shorter than the storm duration are contained in the design storm. It is then possible to read the 5-minute peak intensity directly from I-D-F curves. For the 50-year storm considered above, one obtains the peak intensity from Fig. 4 as $i_{5\text{-min, 50-year}}$ = 190 mm/hr. With other types of temporal distributions, different peak intensities would be obtained (most often lower). This would mean that the 5-minute peak intensity associated with a 6-hour 50-year storm has a different return period than 50 years.

The timing of the peak intensity during the storm is important for runoff computations. Recognizing that loss functions considered

in hydrologic synthesis (e.g., depression storage, infiltration) attain their maximum values at the beginning of the storm, advanced storm profiles will produce lower runoff peaks because the peak intensity coincides with maximum losses. On the other hand, delayed storm profiles produce higher runoff peaks, because the peak intensity occurs in the later part of the storm when losses are low.

The timing $t_r$ of the peak intensity is usually expressed as

$$t_r = t_p/T \tag{5}$$

where $t_p$ is the time to the peak and T is the total storm duration. Thus for a fully advanced storm profile $t_r = 0$, and for a fully delayed profile t = 1. Although the timing of the peak intensity is a probabilistic factor, it is typically considered as a deterministic factor with a constant value. Various approaches to determining $t_r$ have been recommended. In some design storms, the storm profile is assumed to be symmetrical with a centered peak intensity, $t_r = 0.5$ (38). In other cases, the timing $t_r$ is determined by analysis of actual local heavy storms. The timing is then taken as the mean value of all observed timings (25), or as a mean value of timings in the predominant group of storms (53). In the latter case, storms are first classifed into groups with advanced, centered and delayed profiles. After identifying the group with the largest population, the mean value of peak intensity timings is determined for this group and adopted for the design storm (22).

The temporal distribution of storm rainfall completes the definition of the design storm hyetograph. Again, a number of approaches are available and some of these are illustrated in Fig. 6 (33). Design storm profiles are sometimes derived by a government agency from the actual storms and the temporal distribution is then given in Tables (38). The distribution used in the Illinois State Water Survey (ISWS) storm was derived by preparing graphs of the cumulative rainfall vs. time (see Fig. 5) for actual heavy storms and adopting the mean distribution

of the predominant group of storm profiles. In the Chicago storm procedure, the peak intensity timing is determined first and intensities for other durations are positioned around the peak intensity (25).

To demonstrate one of the techniques employed to develop a design storm, an example of developing a simple design storm is given below.

Example of a Design Storm Based on I-D-F Curves (Modified Chicago Storm)

A design storm of the Chicago type is developed below using a procedure which is simpler and more expedient than the conventional Chicago method. In the Chicago method, the I-D-F curves are approximated by a power function, then a storm hyetograph function is derived, discretized and approximated by a step function which is used as a rainfall input in hydrological modelling (32). Similar results can be obtained by deriving directly the step-function hyetograph from I-D-F curves. A detailed description of the simplified procedure follows.

Problem statement: Develop a design storm from the I-D-F curves given in Fig. 3. The storm return period is T = 10 years, the storm duration is 3 hours (= twice the time of concentration assumed as $t_c$ = 1.5 hours), the relative peak timing is $t_r$ = 0.417 (as determined for actual local storms) and the computational time step is 10 minutes. Assume that the design storm contains all 10-year rainfall intensities for durations equal to or shorter than 3 hours.

Procedure: Calculate the peak intensity time as

$$t_p = t_r \times T = 0.417 \times 180 = 75 \text{ min}$$

Thus, the peak 10-minute intensity will occur from t = 70 to t = 80, measured from the beginning of the storm. Note that the peak intensity duration was simply set equal to the computational time step.

By adding one time step on each side of the peak, the next duration is $t_d$ = 10 + 2 x 10 = 30 min. (i.e. from t = 60 to t = 90),

and the following ones are 50, 70, 90, etc. For all these durations, one can determine the corresponding rainfall amounts from the I-D-F curve (T =10 years) and calculate rainfall intensities. Thus, the storm profile is built symmetrically about the peak intensity until the storm beginning (t = 0) is reached. From then on, the remaining time steps are added at the tail end of the storm. An analogous procedure would be followed when dealing with a delayed storm pattern ($t_r > 0.5$) and the end of the storm would be reached first.

Calculations of the design storm hyetograph are shown in Table 5 which is selfexplanatory. The design storm hyetograph is plotted in Fig. 7.

**TABLE 5. CALCULATION OF DESIGN STORM HYETOGRAPH FROM I-D-F CURVES**

| Duration (min) | 0 | 10 | 30 | 50 | 70 | 90 | 110 | 130 | 150 | 170 | 180 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Rainfall depths from I-D-F curves (mm) | 0 | 13.1 | 19.4 | 21.7 | 23.7 | 25.3 | 26.8 | 28.2 | 29.0 | 29.7 | 30.0 |
| Rainfall depth increment (mm) | | 13.1 | 6.3 | 2.3 | 2.0 | 1.6 | 1.5 | 1.4 | 0.8 | 0.7 | 0.3 |
| Rainfall intensity (mm/hr) | | 78.6 | 18.9 | 6.9 | 6.0 | 4.8 | 4.5 | 4.2 | 2.4 | 2.1 | 1.8 |

### 3.5.4.2 Other types of synthetic design storms

A review of engineering applications revealed that various design storms have been applied (not always properly) to many types of drainage design problems, such as sizing of drainage pipes and ditches, comparisons of predevelopment and postdevelopment runoff peak flows, sizing of stormwater storage facilities, control of water quality of drainage discharge, and real-time control of combined sewer systems. In such applications, the storm characteristics to be considered include the total rainfall depth; the storm duration; the frequency of occurrence; the distribution of rainfall during the storm and the corresponding peak intensities for various durations; the peak intensity timing, and the rainfall antecedent to the intensity peak; the antecedent dry period; inter-event time; storm movement (the direction and velocity); and the storm development and decay. All these storm characteristics may affect the results of runoff quantity and quality computations and, consequently, the drainage design. Under such circumstances, the development of design storms is much more complicated than the procedures given in the preceding section (3.5.4.1). Techniques for the development of complex design storms were proposed for specific projects or locations and lack a more general verification. Consequently, such techniques are not included here and the reader is referred elsewhere (23, 26, 32).

### 3.5.4.3 Historical design storms

The discussion in the preceding sections indicates some problems with the development and application of synthetic design storms. Most of these problems concern the assignment of identical return periods to the storm and runoff event, definition of design storm parameters, and definition of antecedent conditions. To alleviate these problems, the use of actual historical rainfall data has been proposed (35). Ideally, one should use a sufficiently long rainfall record to

undertake continuous runoff simulation and to evaluate the drainage system performance. To reduce costs, continuous simulation is sometimes approximated by a series of single-event simulations for selected actual storms (59). The selection of these storms, which effectively replace the rainfall record for a particular purpose, is based on ranking of all storms according to a particular storm parameter. Furthermore, one may have to estimate the initial conditions at the start of individual events and, if required, make adjustments to account for the initial conditions (26). As an example, runoff peak frequency plots for simulations with actual storms in an urban catchment are given in Fig. 8 (32).

The literature on the use of actual storms in drainage design is fairly extensive and a reference is made to several aditional publications dealing with this topic (1, 3, 24, 62, 63).

## 3.6 Water Quality Considerations

When water quality considerations are included in the study objectives, it is necessary to consider the quantity and quality of various inflows to and outflows from the sewer system. Three sources of flow entering the sewer system are considered here - stormwater, dry weather flow, and infiltration and inflow of extraneous water into sewers. The uncontrolled outflow from sewers reaches the receiving waters in two forms - either as discharges of stormwater from storm sewers in separate sewer systems, or as combined sewer overflows from combined sewer systems. The following discussion deals with both the sources of pollution in sewer systems and the outputs from sewer systems.

### 3.6.1 Stormwater quality

It is generally recognized that urban runoff contributes to pollution of surface waters. The magnitude and rate of pollutant emissions in urban runoff are dependent on complex interactions between

climatic processes, land use, and transport in sewer systems. The effect of urban runoff on water quality in receiving waters depends on the magnitude and rate of runoff pollutant emission and the characteristics and uses of the receiving waters. In order to evaluate the effect of urban runoff on receiving waters, it is required to identify both the runoff pollution characteristics and the receiving water characteristics and to establish a cause and effect relationship between the two. This section deals with procedures used to evaluate urban runoff pollution characteristics. It must be emphasized here that the urban runoff quality processes are very complex and not yet fully understood. At the present state of knowledge, the collection of actual water quality data in the field represents the best approach, if time and budget constraints allow this.

The processes controlling quality of stormwater are rather complex, as shown in Fig. 9. The sources of pollution are numerous and their contributions are difficult to quantify. Consequently, the research into runoff quality processes is rather limited and more effort has been devoted to observations of stormwater quality and creation of a data base. Extensive studies of urban runoff quality were reported in Canada (31, 61), Finland (36, 37), France (19), Germany (7, 34), Norway (48), Sweden (15, 30), United Kingdom (29), USA (2, 21, 27, 50, 58), and USSR (41). Caution is advised when transposing runoff quality data to other locations, because the sources of pollutants and their strength are likely to differ from place to place.

Typical concentrations of pollutants in urban runoff, as reported in some German and U.S. studies, are shown in Table 6 (16, 27).

Similar magnitudes and tendencies have been reported in other countries. A general discussion of data in Table 6 follows.

**TABLE 6. POLLUTANT CONCENTRATIONS IN URBAN RUNOFF**

| | Stormwater in Separate Drainage Areas | | | Overflows in Combined Sewerage Areas | | |
|---|---|---|---|---|---|---|
| | U.S.[a] Studies | | German[e] Studies | U.S.[b] Studies | | German[f] Studies |
| Pollutant[c] | Mean Observed Concentration | Stand. Deviation | Mean Observed Concentration | Mean Observed Concentration | Stand. Deviation | Mean Observed Concentration |
| $BOD_5$ | 27 | 25 | 12 | 108 | 36 | 114;156 |
| COD | 205 | 118 | 119 | 284 | 110 | 135;398 |
| S.S. | 608 | 616 | 153 | 372 | 275 | 177;158 |
| Total Coliforms[d] | $3x10^5$ | - | - | $6x10^6$ | - | - |
| Total Nitrogen (as N) | 2.3 | 1.4 | - | 9 | 6 | - |
| Total Phosphorus (as P) | 0.5 | 0.4 | - | 2.8 | 2.9 | 1.4 |

(a) Summary of 20 cities, storm sewers and unsewered areas, ref. 27

(b) Summary of 25 cities, combined sewer areas, ref. 27

(c) All units mg/L except coliforms, MPN/100 ml.

(d) Geometric mean.

(e) Catchment Munich-Pullach (16).

(f) Catchments Munich-Harlaching and Stuffgart-Büsnau (16).

The highest concentrations (~600 mg/L) in stormwater were observed for suspended solids. The main sources of solids include air pollution, soil erosion, wear of solid surfaces,and use of antiskid materials. Suspended solids concentrations in stormwater are higher than those in sanitary sewage.

Biochemical oxygen demand ($BOD_5$) reported for stormwater was rather low (12-27 mg/L). This is caused by low occurrence of organics in runoff. Low observed values of BOD are sometimes caused by interference of toxics with this test.

Chemical oxygen demand (COD) was reported in the range 119-205 mg/L. Note that the COD/BOD ratio for stormwater is much higher than that for sanitary sewage.

Total coliforms were reported as $3 \times 10^5$ - significantly lower than values for sanitary sewage ($10^7$ - $10^9$). Main sources of coliforms are cross-connections between storm and sanitary sewers,and bird and animal feces deposited in urban areas.

Total nitrogen was given as 2.3 mg/L and total phosphorus as 0.5 mg/L. Both these values are rather low compared to sanitary sewage.

It should be noted that many other constituents, some fairly persistent and toxic, have been found in stormwater. Among these, heavy metals are most common with the highest concentrations reported for lead, zinc and copper (12-14). Other substances found in smaller quantities include hydrocarbons, pesticides, and many other toxic substances (30, 45, 65).

Although field observations provide the best indication of stormwater quality, the costs of extensive data collection may be prohibitive in many cases. Consequently, field programmes are typically used either to produce annual pollutant loads for typical urban catchments, or to calibrate stormwater quality models. A brief description of both approaches follows.

### 3.6.1.1 Annual unit pollutant loading rates in stormwater

Annual unit pollutant loading rates in stormwater have been developed in several countries (2, 12, 36, 51) and can be used for an expedient evaluation of pollutant emission in urban runoff. In general, the unit loading rates are expressed as a function of land use and some other factors, such as the annual precipitation, population density, and

street cleaning. In a U.S. study (50), the pollutant rates in stormwater were expressed as

$$M_S = \alpha\ (i,\ j)\ R\ f\ (P_d)\ \gamma \qquad (5)$$

where $M_S$ = weight of pollutant (j) from land use (i) with storm sewers or unsewered per unit areas per year (in ref. 50), the units are lbs/acre/year), $\alpha$ (i, j) is a constant defined for a particular (i, j), R is the annual precipitation, f ($P_d$) is a population density function, and $\gamma$ is the street sweeping effectiveness ($0 < \gamma < 1$). Constants $\alpha$ (i, j) are available for BOD, suspended solids, volatile solids, nitrogen, and phosphorus. For details of application of Equation 5 and numerical values of the parameters $\alpha$ (i, j), reference (18, 50) should be consulted.

Examples of annual unit pollutant loads in stormwater reported in Canada (12) and Finland (36) are given in Table 7. The data reported for Canada were derived from a modified Equation 5.

**TABLE 7. ANNUAL UNIT POLLUTANT LOADS IN STORMWATER**

| Constituent | Annual Unit Loads (kg/ha/yr) | |
|---|---|---|
| | Canada (12) | Finland (36) |
| Suspended Solids | 600 - 1,100 | 100 - 1,000 |
| $BOD_7$ | 40 - 90* | 10 - 100 |
| Total P | 1.5 - 2.2 | 0.2 - 100 |
| Total N | 5.8 - 9.1 | 2 - 10 |
| Pb | 0.2 - 0.3 | 0.1 - 1.5 |
| Zn | 0.6 - 1.0 | - |

*$BOD_5$

### 3.6.1.2 Modelling of stormwater quality

The annual unit pollutant loads which were discussed in the preceding section are useful for a preliminary evaluation of the magnitude of pollution in urban runoff. However, in detailed analyses of water quality in receiving waters, much shorter time intervals (time steps) are considered. Although the selection of such time steps is not well understood, the time steps reported in the practice were as short as five minutes. Such was the case in the study of impacts of combined sewer overflows on English Bay in Vancouver, Canada (9). Short-interval stormwater quality data can be produced by computer models discussed in this section.

Various approaches to the modelling of stormwater quality have been proposed in the literature. These approaches vary from simple regression models to fully deterministic models. The latter models are preferred in practice, because they are transferable to various catchments and require only limited field data for calibration purposes. Among the deterministic models, perhaps the best known are the U.S. models SWMM (Storm Water Management Model of U.S. E.P.A. - ref. 20) and STORM (44, 55). Both models are continuously updated and refined and have been applied in a number of countries. A brief description of some concepts employed in SWMM and STORM follows.

Both models consider the urban surface, particularly the impervious parts, as the most important source of pollutants. During dry weather, dust and dirt accumulate on the catchment surface. The rate of accumulation in time is either linear or non-linear, as specified by the user. The amount of accumulated dust and dirt is reduced by street sweeping, which is assumed to be done at regular intervals and with a certain efficiency. The composition of accumulated dust and dirt is specified by the user, using seven basic constituents - suspended solids, volatile suspended solids, $BOD_5$, COD, Total Coliform, Total N, and Total P). During a storm, pollutant accumulations are washed off the catchment surface at a rate which is directly proportional to the pollutant amount

remaining on the surface. Another important source of solids is urban soil erosion. For this purpose, the Universal Soil Loss Equation (64) is used to quantify the load of suspended solids contributed by soil erosion. The washed-off pollutants are then routed through sewers.

The success of stormwater quality modelling depends on the extent of catchment and calibration data and user's experience. Where only limited stormwater quality data are available, such data are probably best utilized in conjunction with a stormwater quality model.

### 3.6.2 Quality of combined sewer overflows

The composition of combined sewer overflows is generally derived from the composition of the dry weather flow, scour of sewer deposits, and composition of stormwater. Pollutant concentrations in overflows frequently exceed those in stormwater and dry weather flow, because of sewer sediment scour.

The composition of combined sewer overflows has been studied extensively in many countries (7, 27, 29, 31). As an example of pollutant concentrations reported for overflows, some data are given in Table 6 (16, 27).

It is of interest to compare the composition of overflows to that of stormwater. With the exception of suspended solids, pollutant concentrations in overflows exceed those in stormwater several times. The main pollution problems due to combined sewer overflows are those caused by high BOD's, high content of nutrients, and by high bacteria counts which may pose a public health threat.

The earlier discussed techniques for evaluation of stormwater quality are applicable to combined sewer overflows as well. Annual unit pollutant loads in overflows may be calculated from Equation 5 by using proper constants $\alpha$ (i, j). For a listing of these constants, see refs. 18, 50.

The modelling concepts for stormwater are also employed in the modelling of overflows. In addition to runoff sources of pollution, dry

weather flow is considered in quantities and composition specified by the model user. During low flows, sewage sediment is allowed to settle in sewers and may be resuspended during high flows. Pollutant mixing, decay, and routing are also considered (20).

### 3.6.3 Dry Weather Flow - Quantity and Quality

Drainage of many large metropolitan areas is provided by combined sewers. During dry weather, these sewers convey relatively low flows which are referred to as dry weather flows. During wet weather, combined sewers carry dry weather flow as well as surface drainage runoff (stormwater). Because of high runoff inflow, the sewer capacity may be exceeded and the excess flow leaves the sewer system as combined sewer overflow. In order to estimate the composition of flows in combined sewers, one needs to know characteristics of both stormwater and dry weather flow. Stormwater characteristics were discussed in the preceding section and a brief discussion of dry weather flow follows. For a more detailed treatment of this subject, the reader is referred to standard handbooks (60, 40).

#### 3.6.3.1 Quantity of dry weather flow

Dry weather flow comprises liquid and water-carried wastes from residences, commercial establishments, industrial plants, and institutions, together with minor quantities of stormwater and groundwater that are not admitted intentionally.

The best estimates of dry weather flow quantity are obtained by direct flow measurements in the sewer network, or at a wastewater treatment plant. In the absence of such measurements, the quantity of dry weather flow can be estimated from metered water consumption, or calculated on the basis of population serviced with sewerage, population density, land use, tributary area and standard per capita sewage flows.

Apart from domestic wastes discussed above, one also needs to consider industrial wastes. Flows from areas zoned for industrial use will include process flows generated by wet industrial processes and domestic-type wastes from employees. Certain process flows are allowed by municipal ordinance to be discharged into public sewer systems and can affect significantly the quality and quantity of sewage flows in these sewer systems. For modelling purposes, every attempt should be made to collect field data on industrial wastes. When collecting such data on the nature and character of industrial wastes, the following factors have to be considered: the volume of water, distribution in time (the periodicity of flow), the operating schedule of the plant, and the pollutional strength of these wastes. For additional details see ref. 40.

In simulations of flows in combined sewers, variations in sewage quantities during the day and during the week need to be quantified. If no local data on such variations are available, one may use distributions reported in the literature (20). One example of such distributions used in the North American practice is given in Fig. 10. Such distributions are used in conjunction with the locally derived mean flow.

### 3.6.3.2 Quality of Dry Weather Flow

Quality of dry weather flow widely varies from municipality to municipality. It seems to be affected by the population density, the average family income of residents, the use of garbage grinders, industrial wastes and infiltration. The best estimates of dry weather flow quality are obtained by laboratory analysis of sewage samples, or from sewage treatment plant records. In the absence of such field data, estimates of dry weather flow quality are obtained either by tranposition of data from elsewhere, or by calculations based on population and average daily per capita loads. Typical concentrations of various constituents in municipal sewage, as reported in the U.S.A., are listed in Table 8 below (27).

**TABLE 8. POLLUTANT CONCENTRATIONS IN MUNICIPAL SEWAGE (27)**

| | Constituent Concentration (mg/L) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | $BOD_5$ | | COD | | Suspended Solids | | Total Nitrogen | | Total Phosphorus | | To Coli (M |
| | Mean | Range | Mean | Range | Mean | Range | Mean | Range | Mean | Range | Mean |
| Concentration in Municipal Sewage | 200 | 100-300 | 500 | 250-750 | 200 | 100-350 | 40 | - | 10 | - | 5x10 |

* Most Probable Number/100 ml.

Quality of domestic sewage can be calculated from population, average daily loads per capita, and daily per capita sewage flows. In ref. 20, the following average daily constituent loads per capita are given: $BOD_5$ - 0.09 kg/capita/day, suspended solids - 0.1 kg/capita/day, and total coliforms - 200 x $10^9$ MPN (most probable number)/capita/day. Average constituent concentrations can be calculated by dividing these loads by daily flows per capita. Additional loads of constituents are contributed by industrial wastes.

Finally, the strength of dry weather flow is somewhat diluted by infiltration which may be particularly large in older sewer systems.

### 3.6.4 Infiltration and inflow

For the purpose of this manual, the following definitions of infiltration and inflow have been adopted from an APWA report (57):

"Infiltration" covers the volume of groundwater entering sewers and house connections from the soil, through defective joints, broken or cracked pipes, improperly made connections, manholes, walls, etc.

"Inflow" covers the volume of any kind of water discharged into sewer lines from such sources as roof leaders; cellar and yard area

drains; foundation drains; commercial and industrial so-called clean water discharges; drains from springs and swampy areas; etc. It does not include, and is distinguishd from, "infiltration" as previously defined.

Infiltration and inflow are widespread problems adversely affecting most of the existing collection systems and treatment plants. For detailed analysis of infiltration and inflow, the reader is referred to the aforementioned APWA report (57) and WPCF Manual of Practice No. 9 (60). A brief discussion of infiltration and inflow follows.

3.6.4.1 Infiltration

Excessive infiltration may become a serious problem in the design, construction, operation and maintenance of sewer systems. Infiltration was found in the APWA report (57) to be an important source of large volumes of wastewater flow in sanitary and combined sewer systems regardless of the size of the drainage area, types of sewer pipe construction, pipe and jointing materials, and types of soil formations in which the sewers are laid. Infiltration contributes significantly to hydraulic overloading of both collection systems and treatment plants. There are even cases where infiltration and inflow exceed the dry weather flow.

Among the adverse effects of infiltration and inflow, one can name the following:

- cost of treatment of infiltrated water;
- cost of construction of new relief sewers and treatment facilities to accommodate infiltration flows;
- reduced treatment plant efficiency;
- increased bypassing of treatment plants;
- higher pumping costs;
- basement flooding;
- cave-ins and structural failures of sewers and road structures resulting from soil washing into sewers; and
- increased sewer maintenance costs resulting from soil deposits in sewers.

To avoid these costs, some degree of infiltration control needs to be exercised. In new systems, the infiltration control is achieved by adequate design, proper selection of materials of sewer construction, pipes, joints, and proper laying procedures and techniques. Correction of existing sewer infiltration can be accomplished by replacing defective components, sealing the existing openings, and building within the existing component (27). For details, the reader should consult ref. 57. Only a general discussion is presented below.

Infiltration control in new construction

Infiltration control can be achieved by proper consideration of sewer pipe material, selection of sewer joints, and construction consideration.

The important issues to consider in pipe material selection are structural integrity, wastewater characteristics, and local soil or gradient conditions. Combinations of these factors may make one material better suited than another, or preferable under certain conditions (27). Pipe resistance to infiltration should not be omitted from these considerations.

Good sewer joints are probably the most important single factor in the control of infiltration. A good joint must be watertight, resistant to root penetration, effects of soil, groundwater, and sewage, long-lasting, and flexible (27). New jointing methods representing a vast improvement over the traditional cement and asphaltic compounds are described in ref. 27. These methods include PVC and polyurethane joints, compression gaskets and chemical-weld joints.

Major infiltration problems occur in house lateral connections. These connections are made after the sewer pipes have been laid and tested. The connections are made in a crude way and are rarely inspected.

Sewer construction considerations include size of trenches, dewatering of trenchs, pipe laying and assembly, backfilling, and post-

construction performance tests. For details, references 27 and 57 should be consulted.

Infiltration control in existing sewers

The correction of infiltration involves a lengthy systematic plan of action. Such a plan includes the following steps (24):

(1) Identification of the drainage system;
(2) Evaluation of the extent of infiltration (by field measurements, or by examining treatment plant records)
(3) Survey of the sewer system and adjacent soils (smoke tests, soil conditions, groundwater conditions);
(4) Economic and feasibility study to determine the most cost-effective locations for infiltration control;
(5) Sewer cleaning (if required);
(6) Photographic and television inspection; and
(7) Restoration of the sewer system.

Additional details are in references 27 and 57.

3.6.4.2 Inflow

Inflow sources generally represent deliberate connection of drains to a sewerage system. Such connections may be authorized and permitted; or they may be illicit connections made for the convenience of property owners and for solution of on-property problems,without consideration of their effects on public sewer systems (27).

The nature of inflow water and its effect on sewer systems are very much similar to those of infiltration water, as discussed in the preceding subsection.

Correction of inflow problems depends on regulatory and enforcement action on the part of city officials, rather than on

construction measures. The effects of inflows into sewers can be greatly reduced by a variety of methods. For example, many authorities advocate the discharge of roof water into street gutter areas -- or onto lot areas in the hope that it will percolate into the soil (57). Discharging roof or areaway drainage onto the land or into street gutters reduces the immediate impact on the sewer system by providing for reduction of the flow volume and attenuation of the flow. The use of pervious drainage swales and surface storage basins within urban areas allows the storm-water to percolate into the ground (27).

Depressed manholes (those with vented covers in street areas where runoff can pond over the cover) can be repaired or the covers replaced with unvented covers (27).

Commercial and industrial water users should be encouraged to practice on-stream reclamation and reuse of the so-called "clean water" (mostly cooling waters) to control this source of inflows.

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3.8 Figures

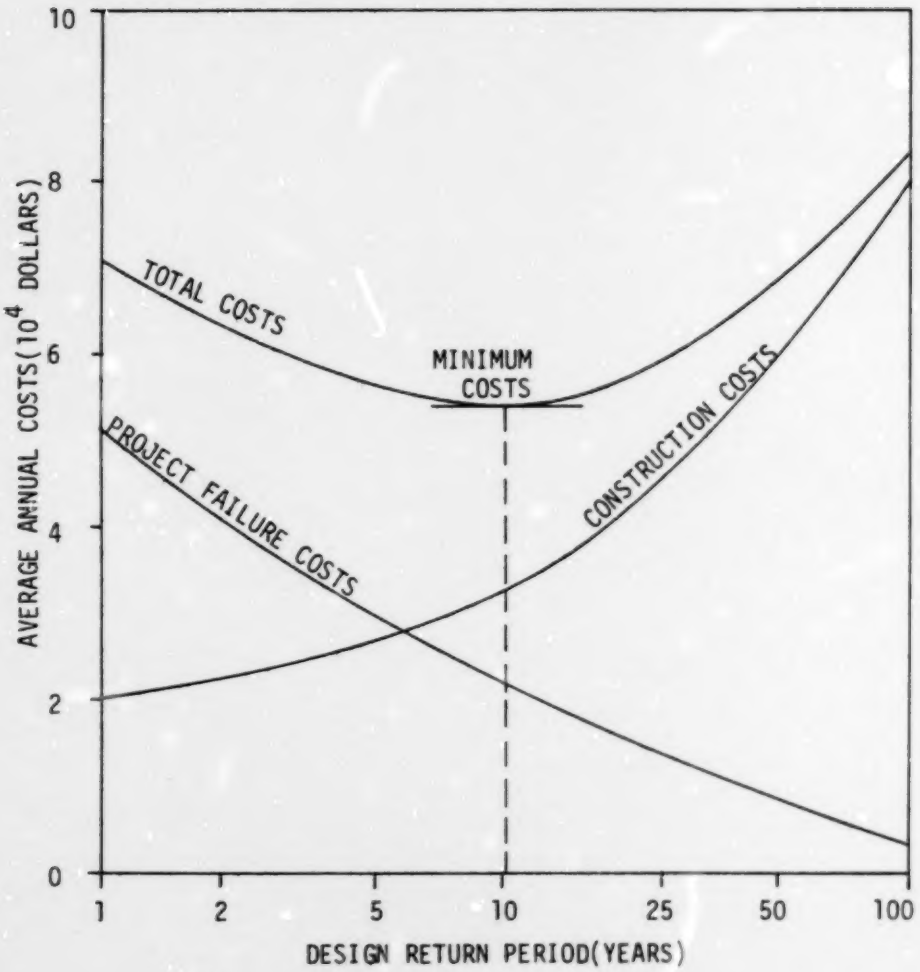


FIG.1. HYPOTHETICAL AVERAGE ANNUAL COSTS FOR VARIOUS DESIGN PERIODS

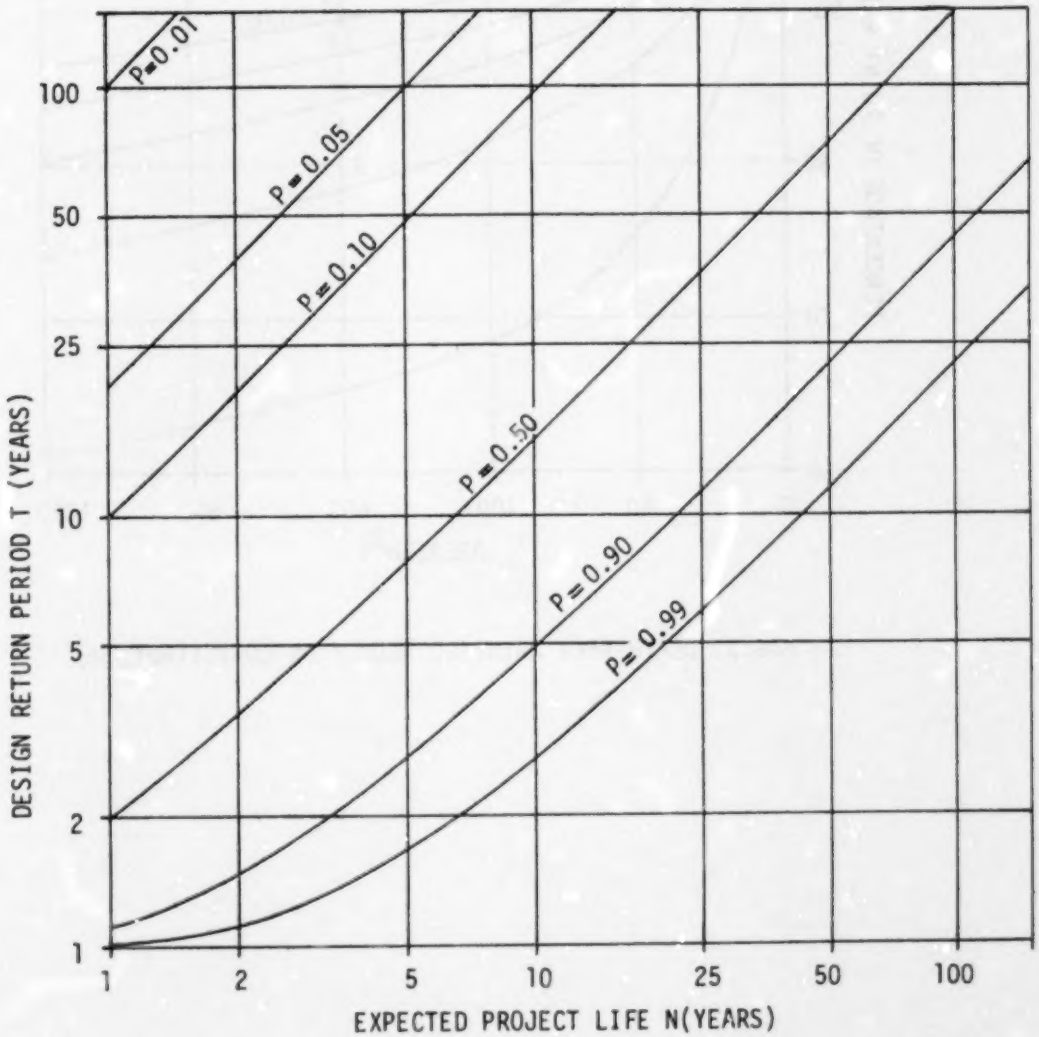


FIG. 2. PROBABILITY OF FAILURE FOR SPECIFIED PROJECT LIFE AND DESIGN RETURN PERIOD

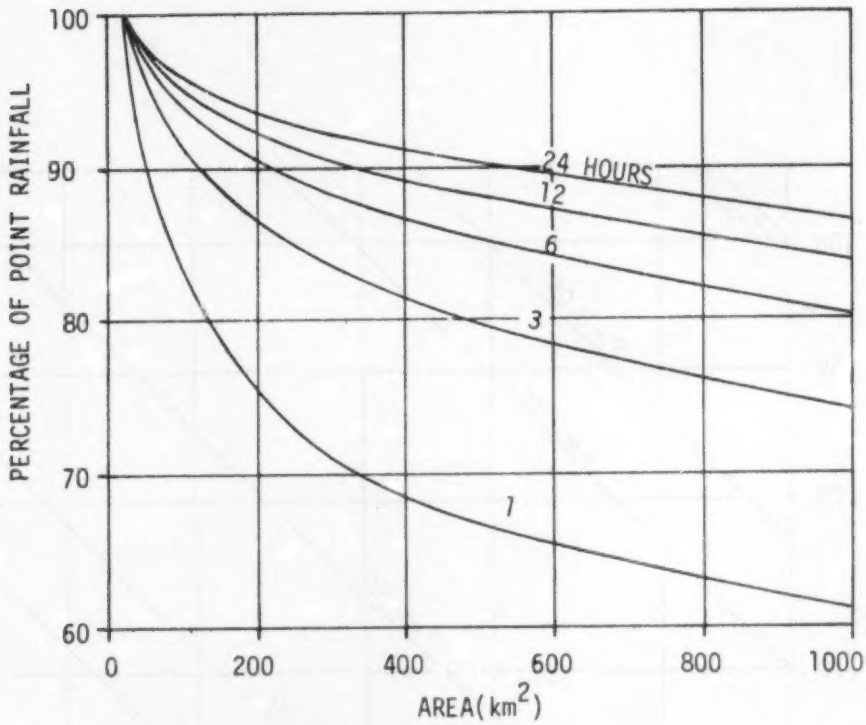


FIG.3. DEPTH-AREA RAINFALL REDUCTION CURVES(Ref.66)

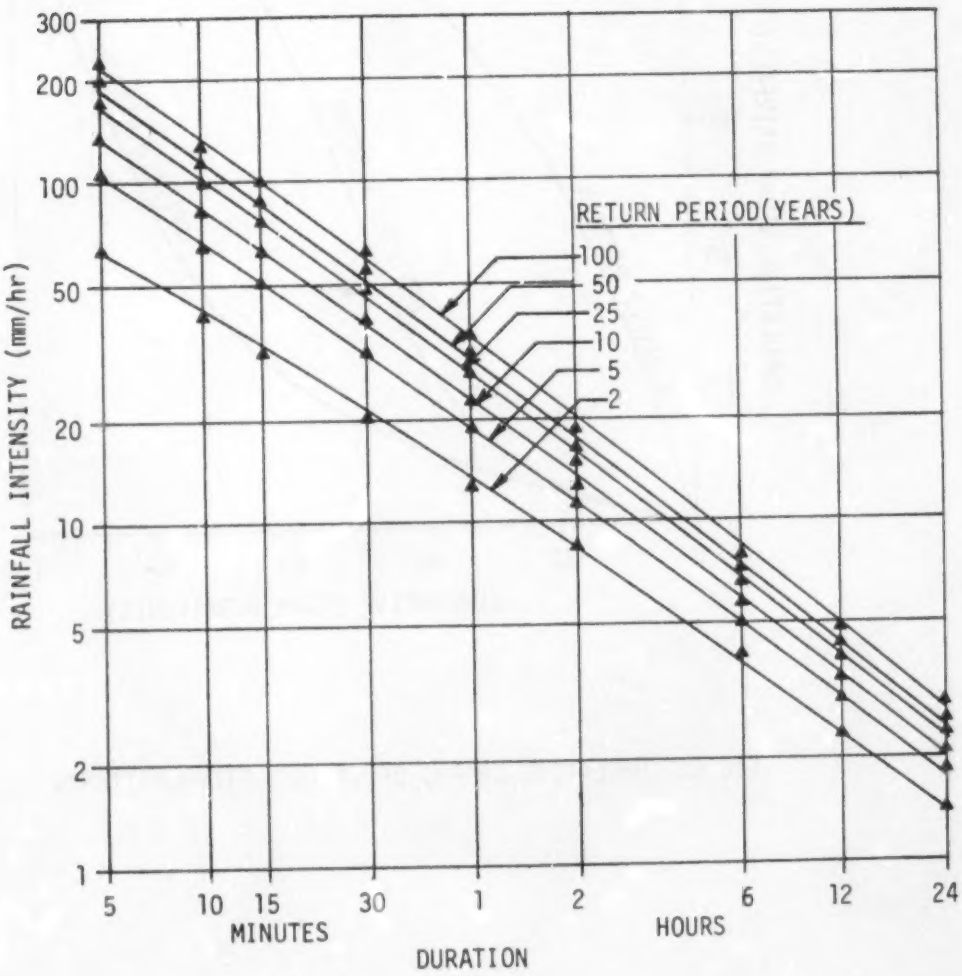


FIG.4. INTENSITY-DURATION-FREQUENCY CURVES

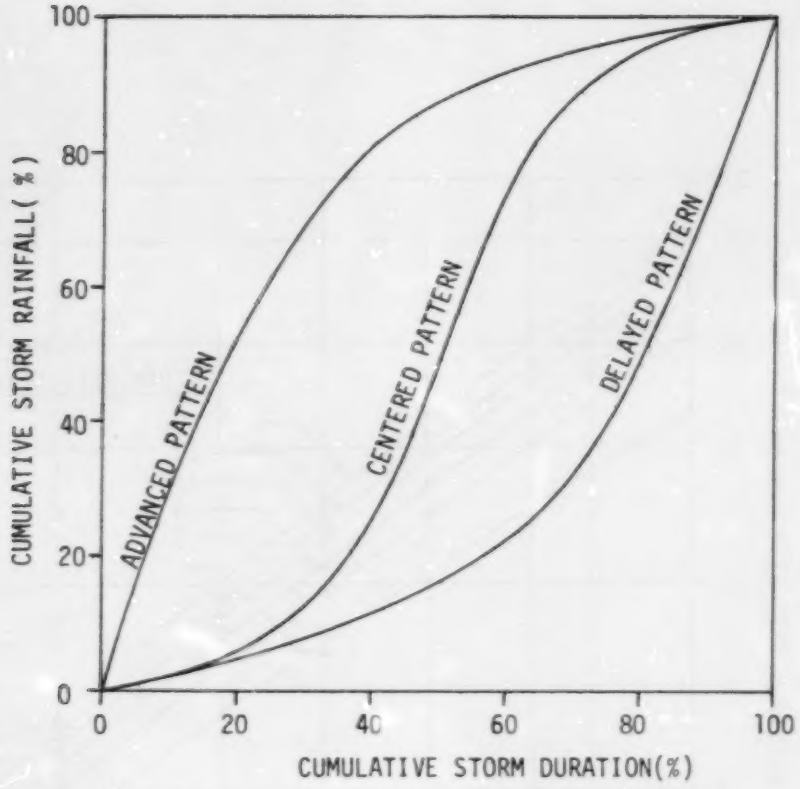


FIG.5. CUMULATIVE GRAPHS OF VARIOUS STORM PATTERNS

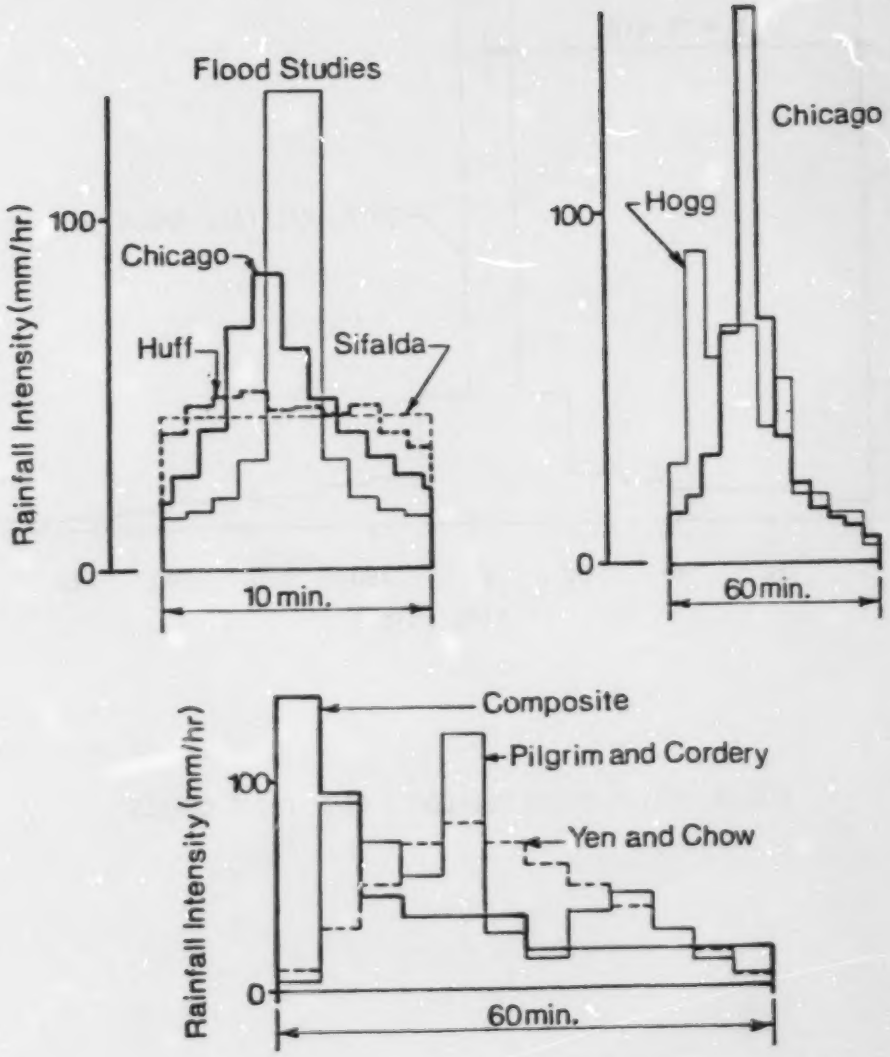


FIG.6. DESIGN STORM HYETOGRAPHS (After refs.3 and 62)

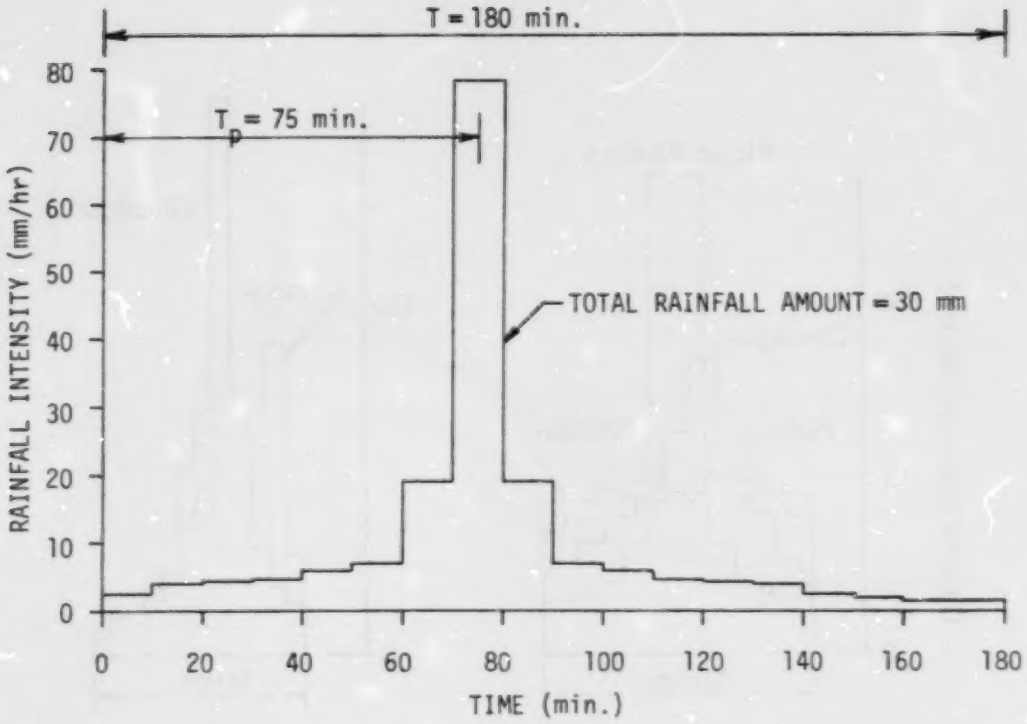


FIG. 7. DESIGN STORM DEVELOPED FROM I-D-F CURVES

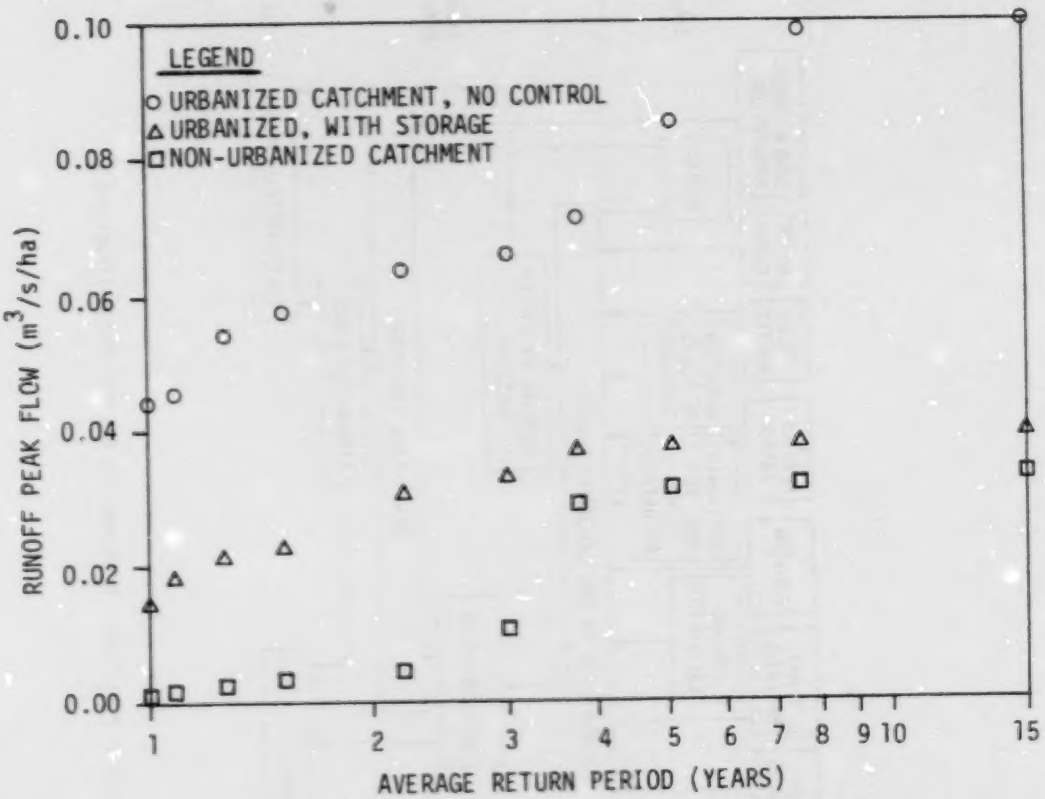


FIG. 8. RUNOFF PEAK FREQUENCY CURVES DERIVED FROM SIMULATIONS WITH ACTUAL STORMS(33)

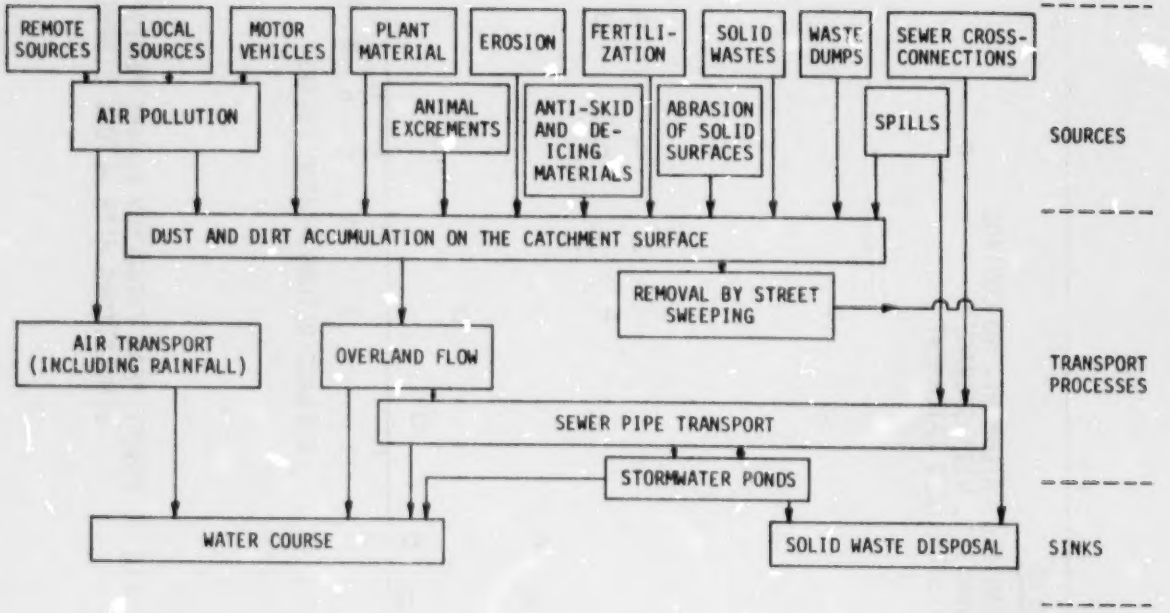


FIG. 9. SOURCES OF POLLUTANTS IN STORMWATER AND POLLUTANT PATHWAYS(12)

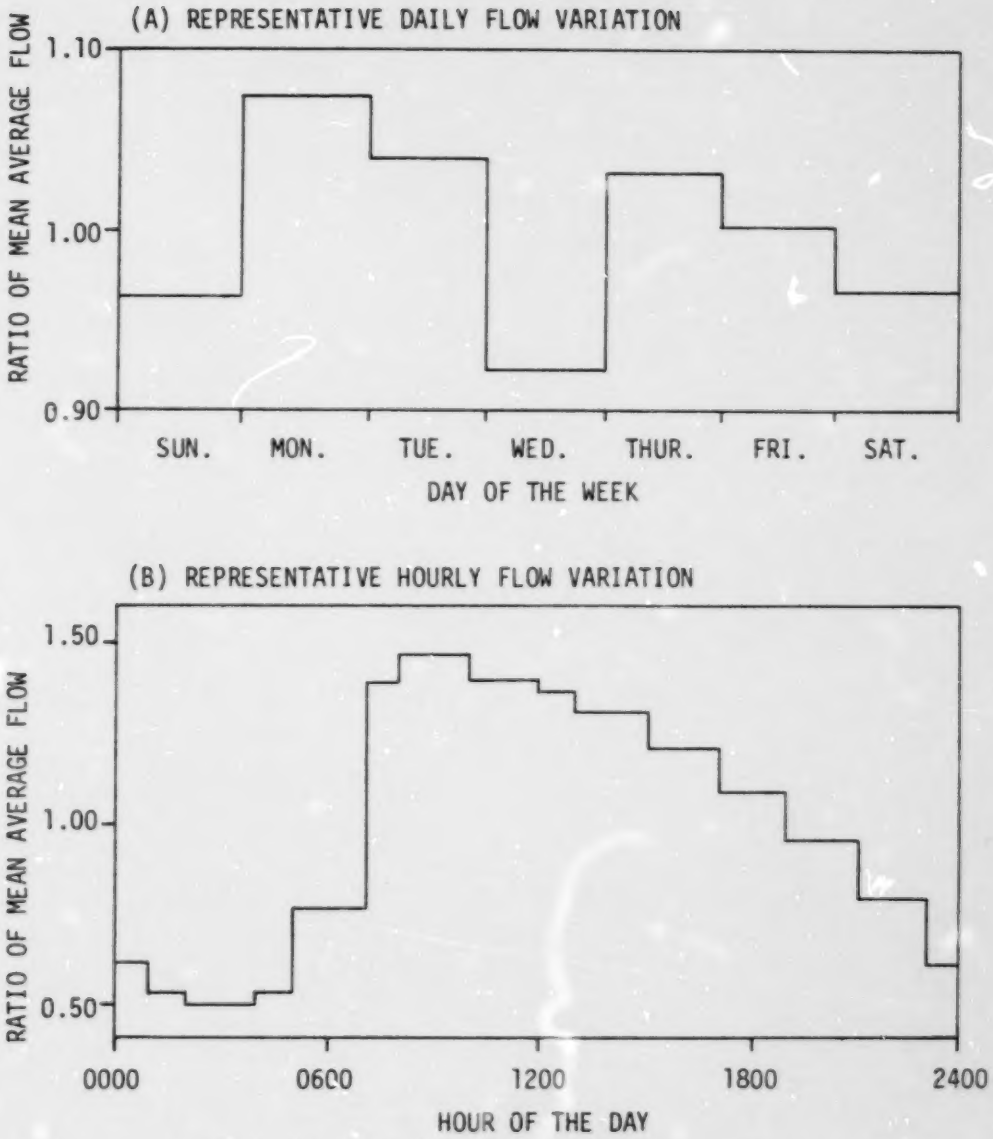


FIG.10. HOURLY AND DAILY VARIATIONS OF DOMESTIC SEWAGE FLOW (20)